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Shake table tests of non-ductile as-built and repaired RC frames

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ABSTRACT: In order to provide information related to seismic vulnerability of non-ductile reinforced concrete (RC) frame buildings, and as a complementary investigation on innovative feasible retrofit solutions developed in the past six years at the University of Canterbury on pre-19170 reinforced concrete buildings, a frame building representative of older construction practice was tested on the shake table. The specimen, 1/2.5 scale, consists of two 3-storey 2-bay asymmetric frames in parallel, one interior and one exterior, jointed together by transverse beams and floor slabs. The as-built (benchmark) specimen was first tested under increasing ground motion amplitudes using records from Loma Prieta Earthquake (California, 1989) and suffered significant damage at the upper floor, most of it due to lap splices failure. As a consequence, in a second stage, the specimen was repaired and modified by removing the concrete in the lap splice region, welding the column longitudinal bars, replacing the removed concrete with structural mortar, and injecting cracks with epoxy resin. The modified as-built specimen was then tested using data recorded during Darfield (New Zealand, 2010) and Maule (Chile, 2010) Earthquakes, with whom the specimen showed remarkably different responses attributed to the main variation in frequency content and duration. In this contribution, the seismic performance of the three series of experiments are presented and compared.

1 INTRODUCTION

Considerable amount of research has been done in the recent past in order to identify structural deficiencies of RC buildings. Theoretical, numerical, and experimental work has been carried out towards developing and empirically demonstrating such concepts, with focus on recommendations for real applications. Due to the lack of knowledge of capacity design principles at that time, introduced only in the 1970s, many issues regarding lack of ductility in RC structures have been identified. In the case of RC frame structures designed according to codes provisions developed before 1970s, several non-ductile detailing was typically adopted, as the use of plane round bars for reinforcement, no stirrups inside the joint, 180° end hooks in beams, lap splices in potential plastic hinge regions, and poor quality of the materials, amongst others (Aycardi et al 1994, Beres et al 1996, Hakuto et al 2000, Park 2002, Pampanin et al 2002).

In the past six years, as part of the FRST (Foundation for Research, Science and Technology, New Zealand) founded project “Retrofit Solutions for New Zealand Multi-Storey Buildings” (www.retrofitsolutions.co.nz), extensive research on feasible, non-invasive retrofit techniques has been carried out at the University of Canterbury. In particular, the experimental research has focused on quasi-static cyclic experimental tests on beam-column joint subassemblies with different configurations and loading protocol (uni- or bi-directional, under constant or varying axial load) in order to understand the local phenomena and damage mechanisms. Several 2D and 3D exterior beam column joints, with and without floor slabs and transverse beams, have been tested, providing valuable confirmation and useful mode in depth understanding of those failure modes observed in actual buildings during past earthquakes. An overview of the project can be found in Pampanin (2009), with more details on the specific topics in Pampanin et al. (2006, 2007a); Marriott et al., (2007); Akgüzel and Pampanin (2008), Kam and Pampanin (2008), Kam (2010), Akgüzel 2010).

As a complementary study of the aforementioned research, related to the dynamic response of RC frame buildings, a 3 storey – 2 bay 3D model structure, composed by 1/2.5 scaled down versions of a prototype frame designed and constructed according to the pre-1970s practice in New Zealand was tested on the shake table facilities of the Structures Laboratory of the University of Canterbury. The original as-built specimen was tested using one ground motion recorded during Loma Prieta Earthquake (California, 1989). Lap splices failure in the exterior columns bottom connections at the third floor level was observed. The specimen was then repaired and modified to avoid lap splice failure, and tested again using data recorded during Darfield (New Zealand, 2010), and Maule (Chile 2010) Earthquakes. Results in terms of observed damage and overall response in terms of inter-storey drift histories are presented herein, with focus on the exterior frame. More information about test results as well as related to the observed damage in the interior frame con be found in Quintana Gallo et al. (2011).

2 SPECIMEN DESCRIPTION

2.1 Geometry

The experimental model was based on a prototype full scale plane frame designed by Marriott et al. (2007). Two of these frames were scaled down to a 40% linear dimension in order to maximize the height and weight of the specimen considering the limitations of the Structures Laboratory facilities, and to fit available materials sizes. These frames were jointed together by transverse beams and floor slabs, adding on one side a slab – transverse beam overhanging to simulate an interior frame. The resulting 3D specimen consists of 2 different frames – exterior and interior, as shown in Figure 1. Specimen main dimensions as well as structural member's sizes are summarized on Table 1.

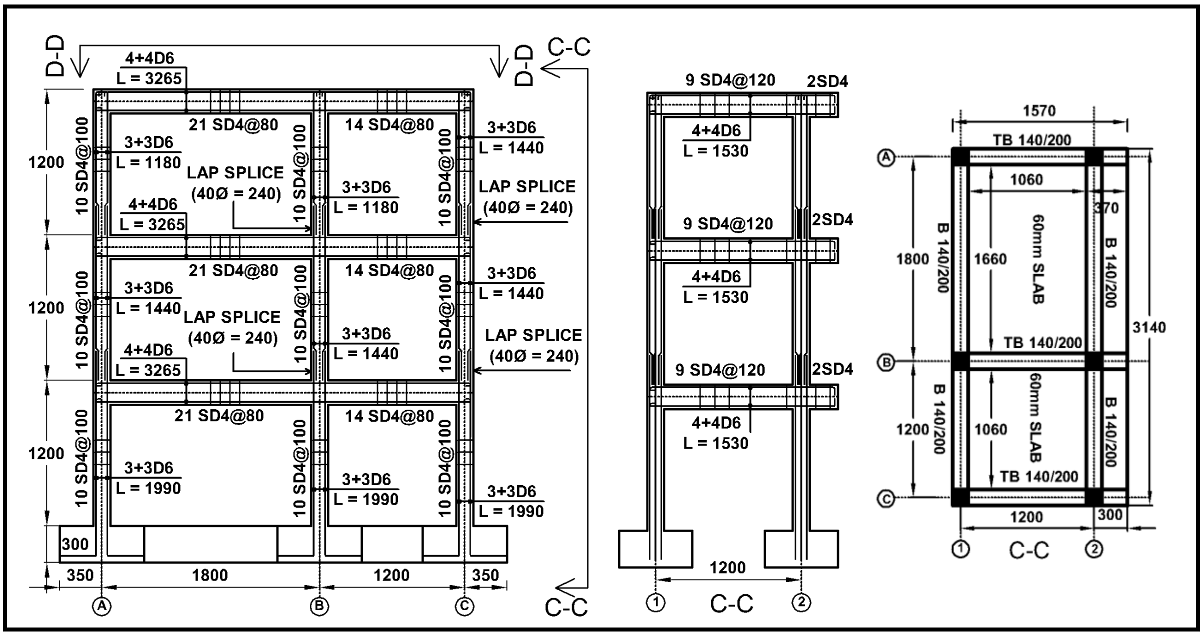

Figure 1: Specimen main dimensions (in millimetres) and reinforcing layout.

Table 1: Specimen main dimensions and member sizes

Dimension	Nomenclature	mm	Member	Nomenclature	Dimension	mm
Total height	**H**	3600	**Longitudinal beam**	**B140/200**	height	200
Inter-storey height	**h**	1200			width	140
Total lenght	**L**	3140	**Column**	**C140/140**	height	140
Total width	**W**	1570			width	140
Long span	**l_1**	1800	**Transverse meam**	**TB140/200**	height	200
Short span	**l_2**	1200			width	140
Orthogonal span	**l_w**	1200	**Slab**	**60mmSlab**	height	60

Following the construction practice, the specimen was casted in-situ (on the top of the shake table). To ensure a rigid attachment to the shake table surface, strong RC strip footings were anchored to the surface using high strength threaded rods. Strips were 300 mm thick and 600 mm wide, reinforced with 10mm corrugated bars. In Figure 2 pictures of the specimen after finishing gross construction and before each test series are presented.

Figure 2: Specimen description: left, after finishing construction; middle, before first test series (as-built specimen); right: before second test series (as-built modified-repaired specimen)

2.2 Reinforcement details

Reinforcement detailing was specified according to 1955 New Zealand code (NZS95:1955, 1955). More specifically, the test frame was characterized by the use of plain round bars, 180° end hooks on beam bars anchorage, lack of confinement/shear stirrups in the joint, lap splices in potential plastic hinge regions in columns, and no capacity design philosophy. Slabs reinforcement was anchored into the beams and slab using 90° end hooks. On the exterior frame, hooks were anchored on the outside beam longitudinal reinforcement, whereas on the interior frame, slab bars were extended from the longitudinal beam to the overhang and anchored into the slab using smaller hook outside lengths. All reinforcement consisted in plain 6mm diameter bars, with the exception of stirrups made of 4mm diameter bars. Details of reinforcement configuration in the panel zone region are shown in Figure 3.

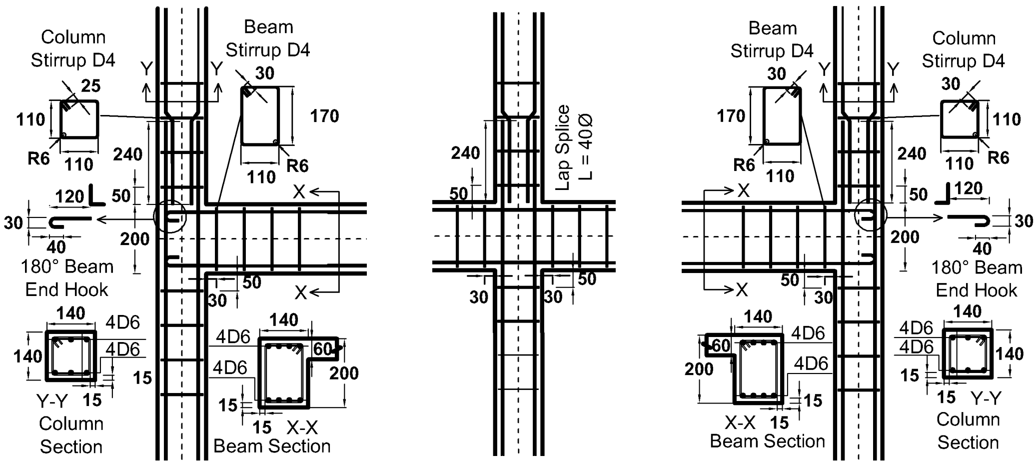

Figure 3: Beam column joint reinforcement details – as built specimen

3 MATERIAL PROPERTIES

Specified strength for longitudinal reinforcement was 300 MPa for 6 mm bars (steel Type 1) and 500 MPa for 4 mm bars (steel Type 2). Both steel types had bigger yielding stresses than specified, as expected. For Type 1, under uniaxial standard tests, strain levels of 10% were reached without loss of strength, and strain hardening was observed at strain levels of 1%. An average yielding stress of 385 MPa and a failure stress of 500 MPa were observed. Steel Type 2 had 585 MPa average yielding stress, without significant strain hardening, and was able to withstand only deformations of the order of 1%. Concrete cylinders were tested at 28 days and during the testing days. Average values obtained for each case are summarized on Table 2, were also the results of compression test of mortar samples for the repaired specimen are presented.

Table 2: Concrete compressive strength values

Concrete compression strength f_c' (MPa)						
Floor	Casting date	28 days	1st test		2nd and 3rd tests	
-	-	f_c'	Days from casting	f_c'	Days from casting	f_c'
1	03-Feb-10	27	184	29	273	30
2	02-Mar-10	23	157	25	246	25
3	01-Apr-10	8	127	11	216	12
Mortar	24-Sep-10	30	-	-	40	33

4 DYNAMIC SCALING

Scaling was done following the rules of dimensional analysis. Among the dimensional numbers incorporated in the problem, Cauchy number requires Young Modulus (E), length (l), and density (ρ) to be related to each other. When using prototype materials, Young's modulus is the same in both domains, requiring an increasing in the density ratio (ρ_r). In order to artificially solve this problem, it was assumed that the additional mass required to reach an artificially increased density is concentrated at each floor level and evenly distributed on each floor level plane. Reactive live load was assumed to be 30% of typical values for residential buildings (2 kN/m^2). As a consequence, 4.3 tons in the form of concrete and/or steel blocks and plates were added on top of the slabs of floors 1 and 2, and 3.0 tons were located on the top level.

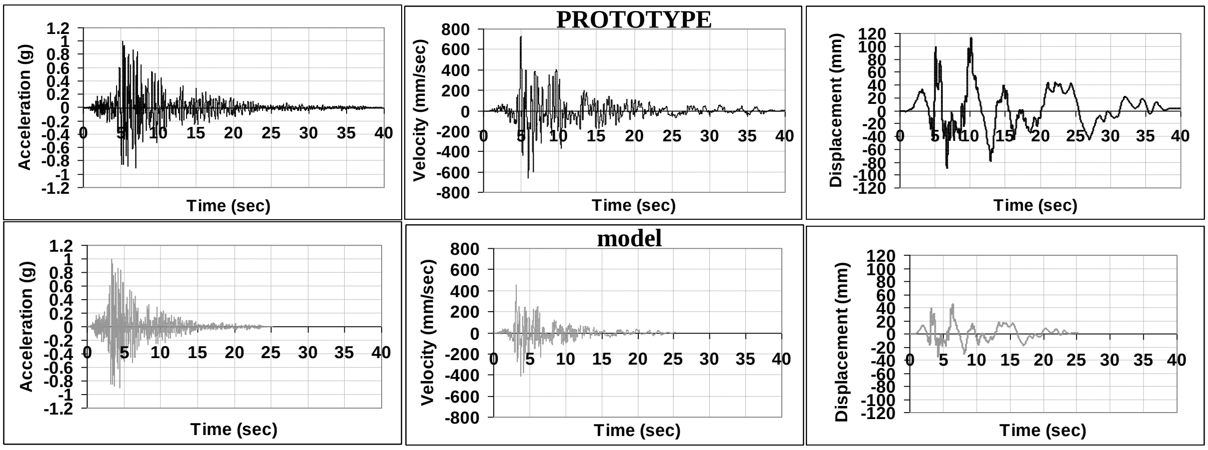

Figure 4: Loma Prieta Gilroy Array #7 record. Left: acceleration; centre: velocity; right: displacement (normalized to PGA =1g)

Input motions were scaled down accomplishing Froude's number, related to acceleration. As the acceleration of gravity was considered in the problem and kept as an invariant between both domains, the acceleration must be the same in prototype and model domains. In order to keep consistency in the three representations of the ground motion, time was scaled down by a time ratio (t_r) equal to $\sqrt{l_r}$, where l_r is the length scale ratio. Loma Prieta Earthquake Gilroy Array #7 station record normalized to PGA = 1g, shown in Figure 4 is used to illustrate the scaling process and rules followed herein. Details can be found in Quintana Gallo et al (2010).

5 INPUT MOTION AND TEST SEQUENCE

Records were obtained from international databases and corrected in order to create a consistent series of ground motion. They are not necessarily identical to the 'true recorded motion' since the resulting acceleration will not match the initial filtered acceleration (Boore 2001, Boore and Bommer 2005). For motion correction, a Band pass, Butterworth 4° order filter was applied to the recorded acceleration in the frequency range of 0.10 – 25.00 Hz. This was done to remove very long period waves that can alter the displacement history, as suggested by Boore (2001). Nevertheless, these corrections do not alter the spectral responses significantly (Boore 2001).

Three different ground motion records were used. They correspond to one horizontal component recorded at a specific station during three different Earthquakes: Loma Prieta (California, 1989), Maule (Chile, 2010), and Darfield (New Zealand, 2010). A summary of main characteristics of the records is given in Table 3. In Figure 5 the scaled acceleration time histories used for each test series are presented. Tests sequence is summarized in Table 4, where SF means the ratio between original on site recorded PGA and testing PGA. In a first test series (1.1, 1.2, 1.3), Gilroy Array #7 (Loma Prieta Earthquake) was used. It was run three times with increasing nominal PGA values: 0.45g, 0.68g, and 0.9g. After the third test this series was ended. In a second series of testing (2.1, 2.2, 3), corresponding to the modified – repaired as-built model building, Christchurch Hospital record (Darfield Earthquake) was used at PGA levels of 0.17g and 0.20g in a first instance. Lastly, Marga-Marga record (Maule Earthquake) at PGA = 0.34g was used.

Table 3: Earthquake summary and station location

Event	Date	Country	M_w	Depth (km)	Station	Region	R (km)	PGA (g)
Loma Prieta	Oct. 17th 1989	USA	6.9	-	**Gilroy # 7**	California	24	0.21
Darfield	Sep. 4th 2010	NZ	7.0	5	**ChCh Hospital**	Christchurch	45	0.20
Maule	Feb. 27th 2010	Chile	8.8	35	**Marga-Marga**	Viña del Mar	290	0.34

Table 4: Test sequence

Test #	Specimen	Record	SF	PGA (g)	Duration (s)
1.1	as built	Gilroy Array #7	2.1	0.45	25
1.2	as built	Gilroy Array #7	3.2	0.68	25
1.3	as built	Gilroy Array #7	4.3	0.90	25
2.1	as built - repaired	ChCh Hospital	0.85	0.17	40
2.2	as built - repaired	ChCh Hospital	1	0.20	40
3	as built - repaired	Marga-Marga	1	0.34	65

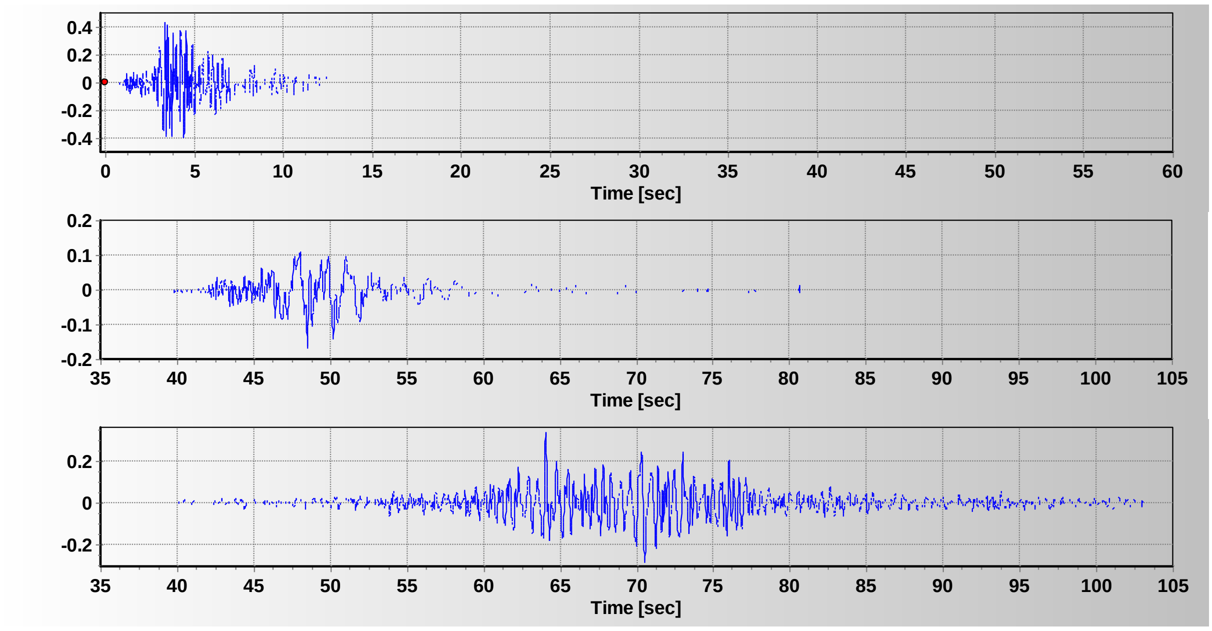

Figure 5: Gilroy Array #5 record from Loma Prieta Earthquake (California, 1989)

In Figure 6 the acceleration and displacement response spectra are shown for the three records in the model domain. This means, the period is reduced by a factor of √0.4, the displacement by a factor of 0.4, whereas the acceleration remains equal. Also plotted for comparison are the New Zealand design spectra for different soils and a probability of exceedence of 2% in 50 years, for the city of Christchurch (Z = PGA = 0.22g). Also for reference, according to the NZS1170.5 specification, a spectrum generated for Christchurch with a 2% probability of exceedence in 50 yrs (approx. 2500 years return period) corresponds to a spectrum generated for Wellington with a probability of exceedence of 10% in 50 yrs. (Z = 0.40, approximately 500 years return period).

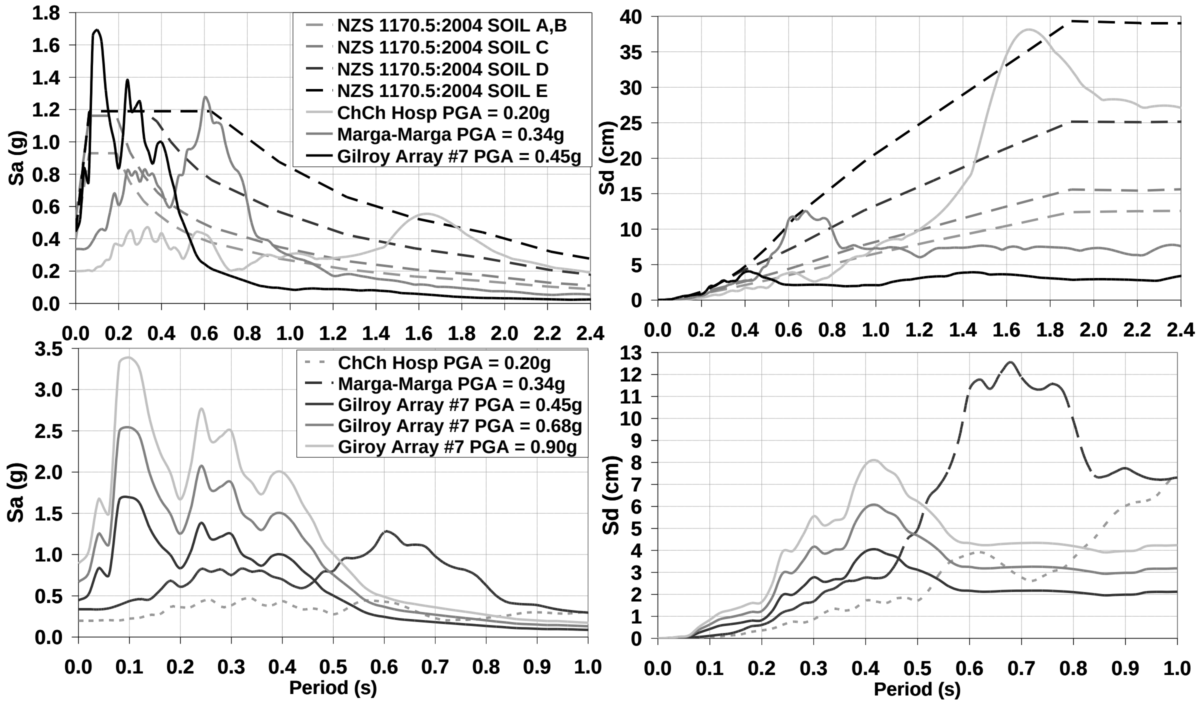

Figure 6: Response spectra comparison

6 LOMA PRIETA TEST RESULTS

Tests 1.1, 1.2 and 1.3 were conducted in a consecutive sequence, increasing the amplitude of the initial input motion - PGA = 0.45g, Figure 5 - by a factor of 1.5 and 2.0. During Tests 1.1 and 1.2 (thus up to PGA=0.675g) a predominantly elastic response was observed. No evident damage was developed in the structure. During Test 1.3 though (PGA=0.9g), a response characterized by excessive lateral deflections of the top floor, almost disconnected from the floors below, was observed. Recorded inter-storey drift time histories for Test 1.1 and 1.2 are presented in Figure 7. In Figure 8 recorded inter-storey drift for Test 1.3 as well as relevant observed damage on the exterior frame are shown. As can be seen in Figure 7, for Test 1.1 at PGA = 0.45g, inter-storey drifts remained below 1%, being recorded time histories almost identical in all floors. For Test 1.2 at PGA = 0.6g, no evident increasing in the drift levels was observed at floor one and two, with an increase of the top floor maximum drift level to 1.5%. Time histories responses showed different dynamic response to what observed in Test 1.1, with drift level being fairly similar in all floors. In the case of Test 1.3, top level drift reached a maximum value of approximately 2.5%, whereas drift levels in floors 1 and 2 remained below 1.5%. Experimental time histories reflect a response governed by a top floor storey local mechanism.

Crack pattern indicates that most of the inelastic behaviour occurred at the base of columns affecting the panel zone region. In the columns at the third floor, horizontal cracks developed just above the joint or with little strain penetration, whereas on the second floor, horizontal cracks penetrated significantly inside the panel zone. Vertical cracks and diagonal cracks developed on exterior joints, on both faces. Crushing of concrete was observed on the bottom of third floor columns. This reflects rocking action at the bottom of exterior columns due to loss of bond between concrete and reinforcement in column lap splices. On top floor joints diagonal cracks developed in the opposite direction to those of floors 1 and 2. In this case, cracks developed following the strut resulting from the beam acting with the slab in compression, whereas on the other, cracks tend to be oriented following the compression strut generated when the slab acts in tension. As can be seen in pictures of Figure 9, when bond is lost in the exterior longitudinal bars of the column, tension capacity is lost in that area. On the interior bars, whose lap splices apparently did not fail, a compression strut was able to be developed in the joint. On top joints on the other hand, the compression strut is developed as expected.

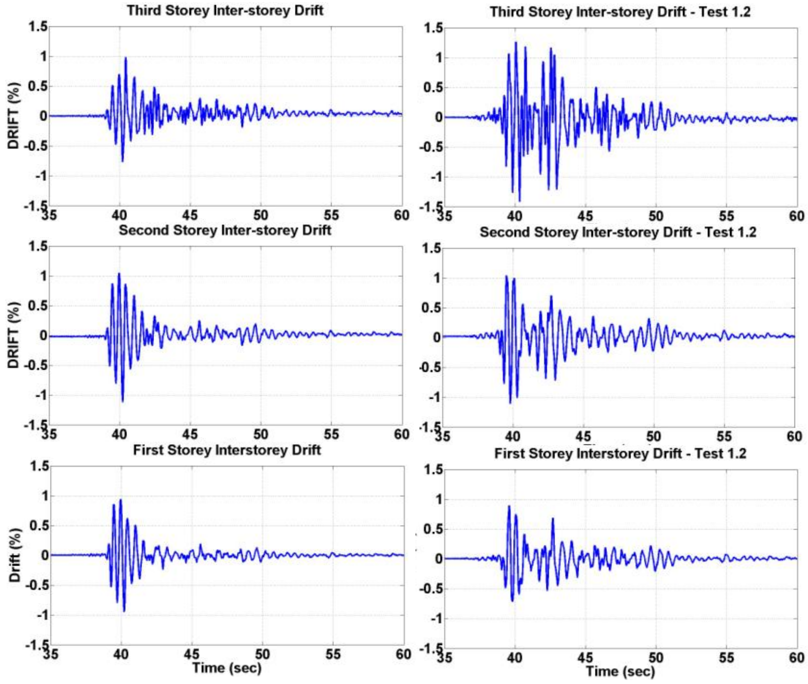

Figure 7: Recorded inter-storey drifts Tests 1.1 (left) and Test 1.2 (right)

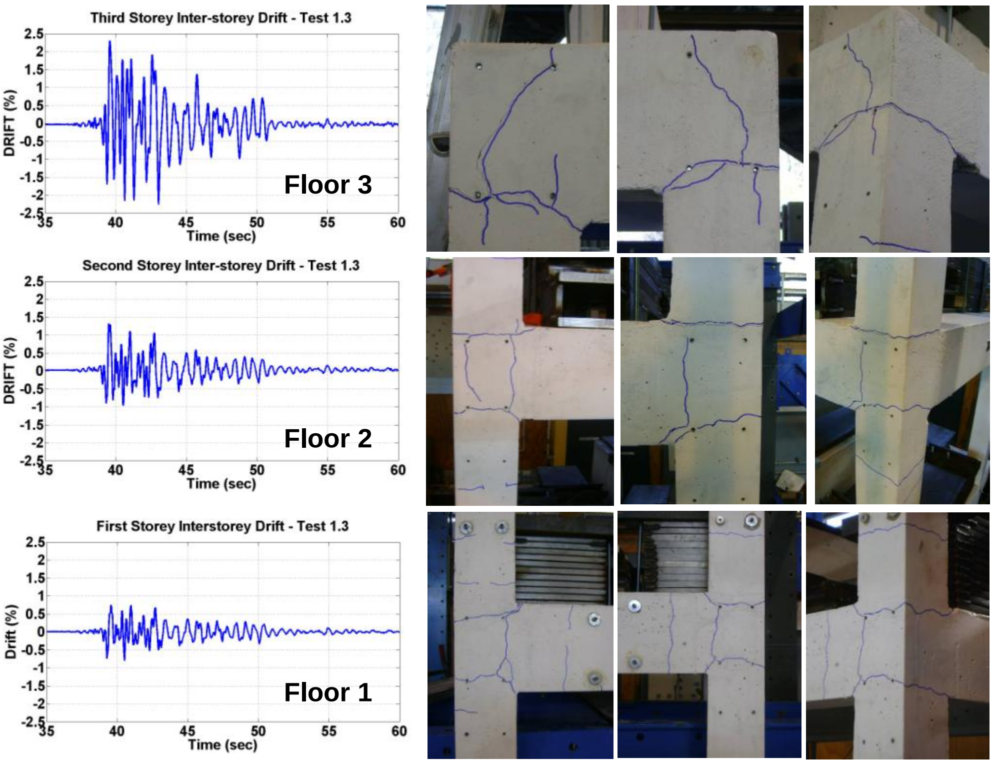

Figure 8: Recorded inter-storey drifts and observed damage – Test 1.3 (Loma Prieta)

The failure mode described in previous paragraphs was corroborated with the observed damage in the specimen after the Darfield Earthquake that stroke the city of Christchurch on February the 4th 2010. As shown on the pictures of Figure 9, vertical and diagonal cracks in the same direction as observed before on the left corner joint were developed on the right corner joint. Rocking on the bottom of third floor column was clearly reflected by the crushing of the concrete around the column reinforcing rebar. The bi-directional characteristics of the real ground motion can be appreciated in corner beam column joints, where a symmetrical damage pattern was observed. It is important to clarify that, since the actual ground motion affected the specimen in real time, even if the input acceleration is the same in prototype and model domains, time does not accomplish similitude rules. As a consequence the response in terms of displacements, velocity and floor accelerations are much bigger to an equivalent response under the same similitude-compatible input motion.

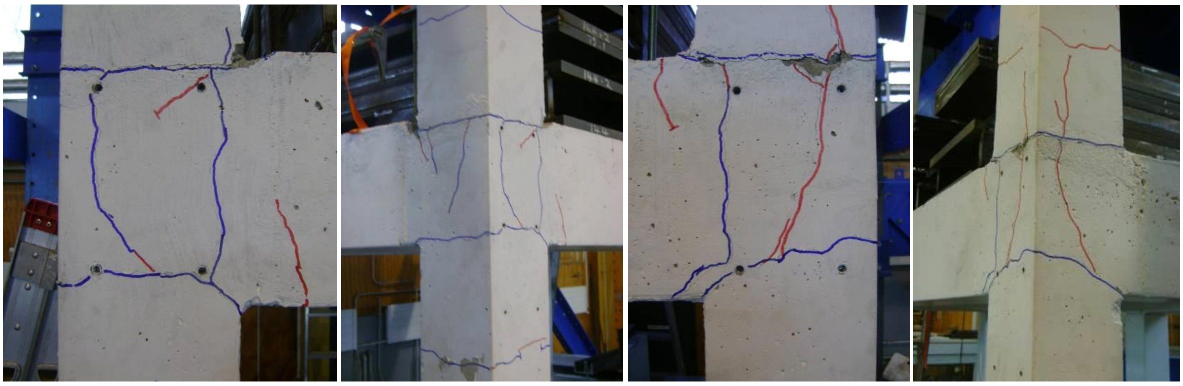

Figure 9: Observed damage after the real Darfield Earthquake (4th February 2010); new cracks in red.

7 REPAIRING PROCESS

After completing the first testing series, and following the "unplanned" test under the real Darfield Earthquake, the specimen was structurally modified and repaired in order to overcome the structural weaknesses which led to a rocking mechanism of the third floor, namely lap splice failure and lower than anticipated concrete strength at the third floor (11MPa, see Table 2). The concrete around all columns of second and third floors lap splice region was first removed. The longitudinal column reinforcement was then welded along the lap spice to provide continuity in the reinforcement. The removed concrete was replaced with structural SIKA Monotop Structural Mortar, expected to have similar mechanical characteristics in terms of compression strength and adherence. All cracks were filled by injecting SIKADUR 52 strong epoxy resin, with high pressure bombs. Finally, some parts were finished using SIKA 31 epoxy. The new specimen thus retained same dimensions and reinforcement showed earlier in this paper, with the exception of column reinforcement, becoming now continuous in height without lap splices. In Figure 10 repairing intervention is presented schematically. Details of the repairing process can be found in Quintana Gallo et al (2011).

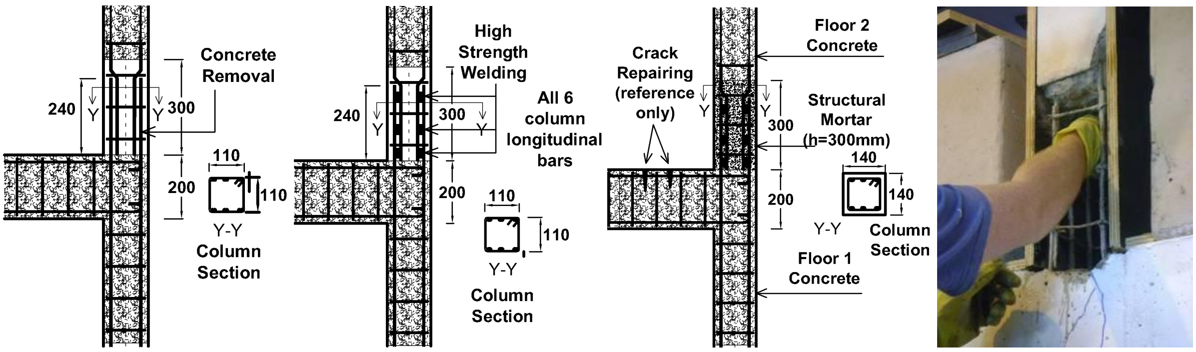

Figure 10: Repairing sequence: concrete removal – reinforcement welding – mortar filling

8 DARFIELD RESULTS

After the specimen was repaired, the ground motion recorded during Darfield Earthquake at Christchurch Hospital was used for Test 2.1 and 2.2. After Test 2.1 at PGA = 0.17g, few very thin cracks were observed along the structure. After Test 2.2, with a slight increasing in the PGA to match the actual recorded PGA of 0.20g, some light cracks developed around the panel zone region and beams, but the structure practically responded in the elastic (pre-yielding) range. Crack patter after the test is shown in the pictures of Figure 11, as well as recorded inter-storey drift time histories at each level.

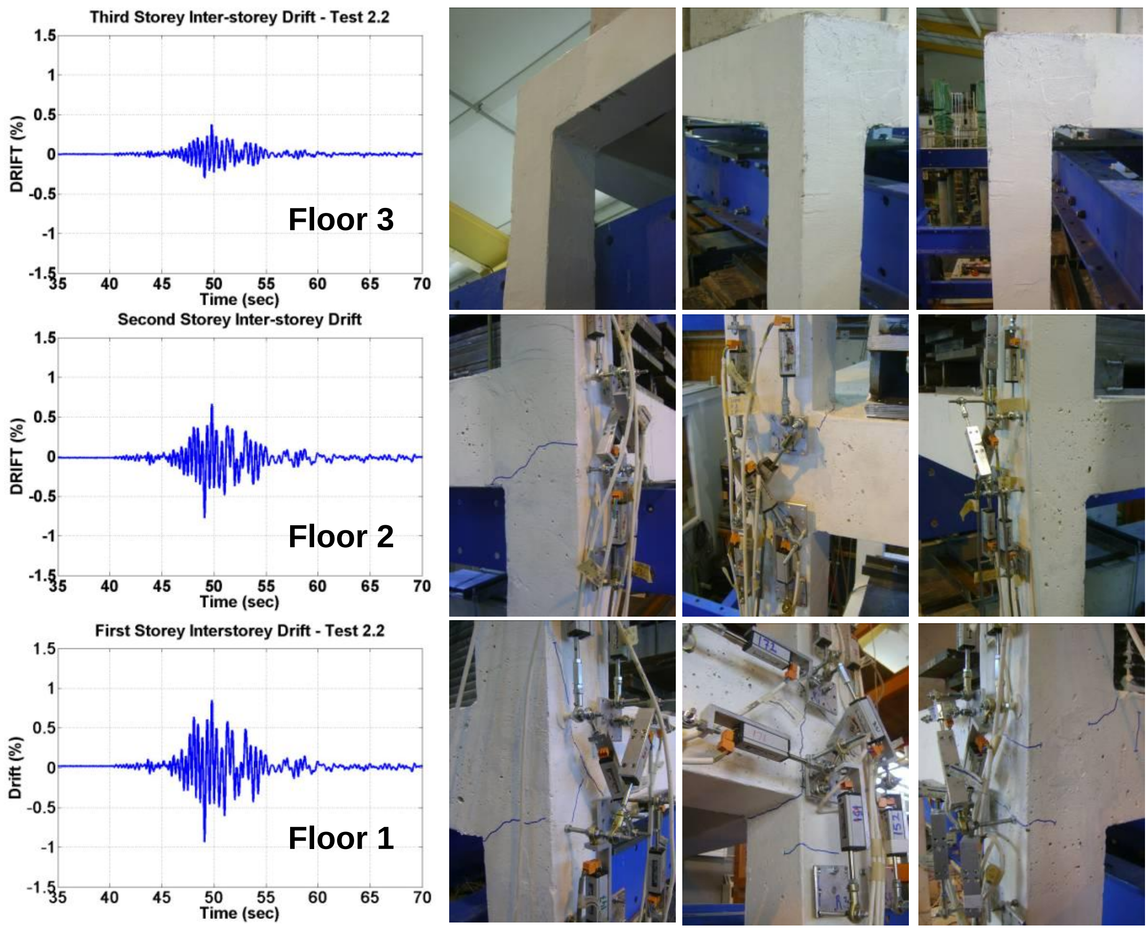

Figure 11: Damage patter observed after Christchurch Hospital record at PGA = 0.20g (as recorded)

Inter-storey drifts remained below 1% in floors 1 and 2, while at the top floor remained below 0.5%. The shape of the response is mainly identical in all floors, differing only in the amplitude, which decays in upper floors. The response in terms of maximum inter-storey drift values is similar to that recorded during Test 1.3, even though that particular input record had a nominal PGA two times bigger than the one used in Test 2.1. This indicates and confirms that the response in terms of displacement is not necessarily bigger for a ground motion with bigger PGA.

9 MAULE RESULTS

After the specimen was tested two times under the record corresponding to Darfield Earthquake, a last test was conducted using data recorded at Marga-Marga station (Viña del Mar city) during the Chilean Maule Earthquake (February the 27th 2010). The intention was to simulate a higher magnitude (8+) and long-distance (approx.150 to 300 km; from Hokitika to Haast) thus long duration earthquake ground motion, as it would be the one generated by the Alpine Fault, which still represent the main contribution of the seismic hazard for Christchurch. Results in terms of recorded inter-storey drift time histories and observed damage are presented in Figure 12. As can be seen in the pictures, corner joints in the first floor suffered severe damage, and developed the so called ‘concrete wedge’ as a crack pattern. Crushing of concrete in the centre of the joint was also observed. Some cracking was also observed in columns and beams close to the panel zone region. On corner beam column joints of second floor, diagonal cracks were developed, with smaller residual cracks than those measured on the first floor, and no significant crushing of concrete. A maximum inter-storey drift of 4% was recorded on the first floor, whereas a maximum of 2.5% was observed on second floor. In the top floor no significant damage was observed, and a maximum inter-storey drift of 0.4% was recorded. Observations indicated that a first soft storey mechanism was developed, with the structure withstanding a near collapse limit state. Forensic assessment of the building revealed that all for exterior beam column joints of the first floor were severely cracked inside.

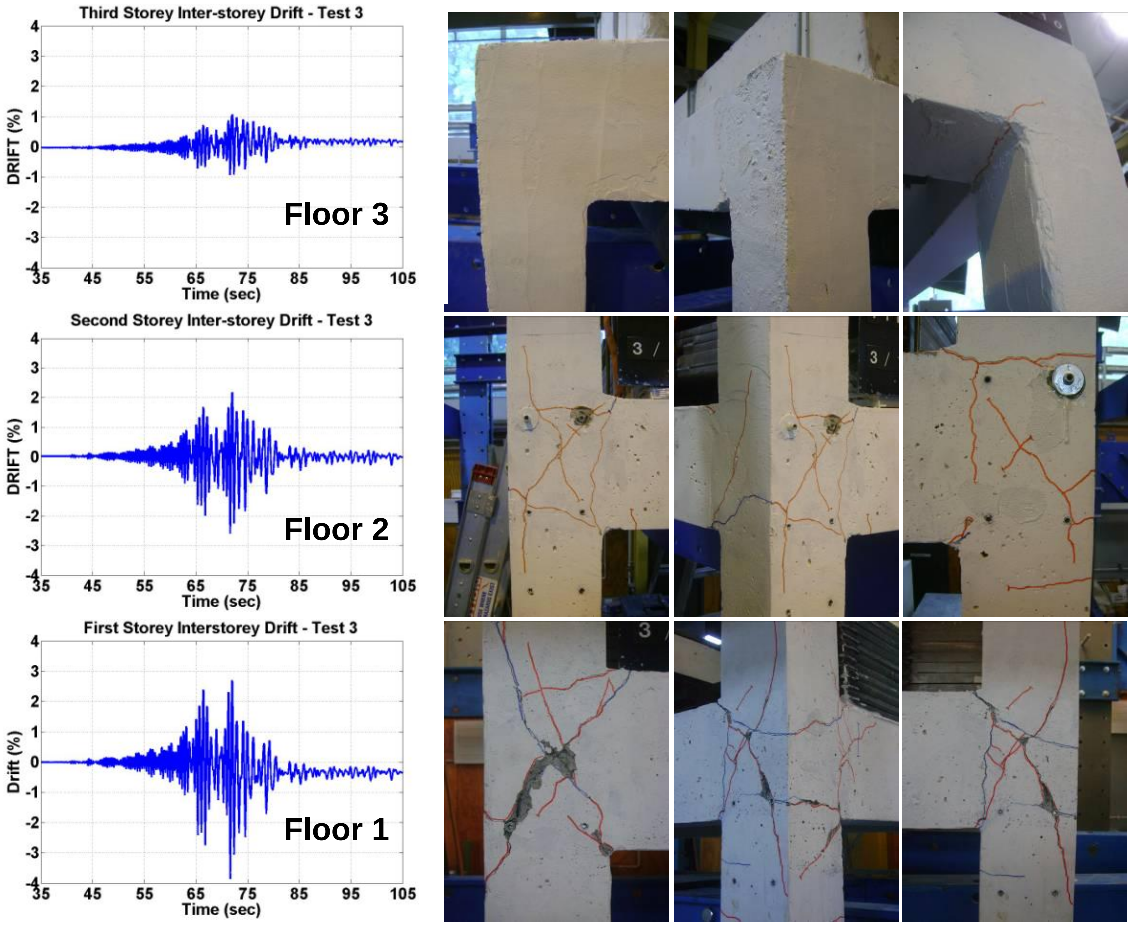

Figure 12: Inter-storey drift recorded time-histories, Marga-Marga record, Maule Earthquake at PGA = 0.34g (as recorded)

10 PRELIMINARY CONCLUSIONS

Three series of shake table tests were performed to the model structure presented in the previous paragraphs. The first series corresponding to the as-built initial specimen was conducted using Gilroy Array #7 record, from Loma Prieta Earthquake at different levels of PGA (0.45g, 0.68g, 0.9g). Observed and recorded response indicate that the structure remained mostly in the elastic range during Tests 1.1 and 1.2, whereas a lap splices failure mechanism developed in the base of the columns of the top floor during Test 1.3, being the overall response controlled by an autonomous rocking of the top floor. This was mainly attributed to loss of bond between smooth plain round bars and very low strength (thus low bond) concrete (11MPa on the third floor). After this first test series, the specimen has been unexpectedly excited by the real Darfield Earthquake, which attacked the Structures Laboratory of the University of Canterbury on September the 4th 2010. This ground motion, corresponding to a much bigger excitation from the model domain perspective than a similitude-consistent record, further developed the failure mechanism observed previously in Test 1.3, adding naturally the complete tri-dimensional response of the structure which is reflected in the damage pattern observed. Right after these test series, the specimen was modified and repaired in order to overcome the aforementioned construction weaknesses and capture a different failure mode. For that, in the lap splice region, the concrete was removed and all longitudinal column bars were welded to provide continuity along the height. The removed concrete was then replaced with structural mortar and cracks filled with epoxy resin.

Considering the very valuable information gathered from recognissance trip and reports from both the Maule (Mw 8.8, Chile, 2010) and the Darfield (Mw 7.0, New Zealand, 2010) Earthquakes records from both events were used as input for a second stage of the shaking table tests. The specimen tested under the ground motion recorded at Christchurch Hospital suffered little damage in the form of thin cracks developed in the panel zone, columns and beams, when subjected to Test 2.2 (PGA = 0.20g – as recorded on site), confirming the relatively little level of damage observed in RC buildings (including pre-1970s) in the city of Christchurch (Kam et al., 2011). The recorded inter-storey drifts remained in fact below 1% in all floors, and were almost identical, indicating a fairly elastic (first mode) response. Given that result, the specimen was then subjected to a strong ground motion recorded at Marga-Marga station, located in Viña del Mar, during the Chilean Maule Earthquake (Test 3 at PGA = 0.34g – as recorded on site). In this case, severe damage was observed on beam column joints on both exterior and interior frames (for information on the interior frame see Quintana Gallo et al 2011). Diagonal cracks of considerable width were developed in both corner beam column joints on the first floor with crushing of concrete in the core, as well as crushing in the bottom of columns. In second floor lighter damage was observed, mainly in the way of diagonal cracks in corner beam column joints. On the other hand, almost no damage was developed in the third floor. Inter-storey drift reached a maximum level of 4.0% in the first floor, 2.5% in the second, and remained below 1.0% in the top floor, consistent with the observed damage patter.

When comparing all tests, many questions arise about the uncertainly inherent to the response of structures under seismic actions. Firstly, the role of PGA as a seismic hazard parameter clearly appears to be overestimated since it does not seem to provide by itself a good correlation with the expected damage that a structure may experience. Frequency content as well as duration of the input motion are confirmed to be evenly if not more important parameters when evaluating the expected performance in terms of damage. Secondly, records response spectra used in the experiments described herein reveal the importance of considering displacement spectra for the design and assessment of structures within a performance based approach. What can appear as a severe motion in terms of spectral accelerations can instead demonstrate to be a relatively low demanding event in terms of spectral displacements and vice versa. As shown in the case of Christchurch Hospital record from the Darfield Earthquake, the spectral displacements are quite low in the short period range, being unable to highly excite structures with fundamental period typical of RC buildings. The Gilroy Array #7 from the Loma Prieta event shows extreme spectral acceleration demands, also shifted up by means of increasing PGA levels, but rather moderate corresponding spectral displacement even at peak level. Finally in the case of Marga-Marga record from the Chile Earthquake, even if the PGA values were not as big as those of Gilroy Array #7 Tests 1.1 and 1.2, the observed damage was substantially bigger and drift levels reached are close to those associated to a near collapse limit state. This is attributed to a different frequency and energy content of the record with a clearly superior duration (number of input cycles), leading to more severe inelastic excursions of the model structures, associated to stiffness and strength deterioration of brittle component as the joint areas..

11 ACKNOWLEDGEMENTS

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